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Innovations and performances of PVD and dual function geosynthetic applications

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Abstract

Full-scale test embankments, which were constructed on separate locations but within the site of the Second Bangkok International Airport and at the Campus of the Asian Institute of Technology, confirmed that prefabricated vertical drain (PVD) installation not only accelerated and facilitated uniform consolidation settlements but also aided in recharging the subsoil. Thus, PVD can mitigate the negative piezometric drawdown of the subsoil caused by the excessive pumping of groundwater down to the depth of PVD installation. The compressibility and flow parameters were back-calculated from the performance of the full-scale tests. Further back-analysis proved that every stage of preloading corresponded with certain rate of consolidation, and hence the variation of compressibility parameters of PVD-improved soft clay. Another full-scale test embankment employing vacuum-assisted preloading revealed that the rate of settlement has increased by 60% and the period of preloading was reduced by 4 months. Moreover, the use of electro-conductive PVDs could further shorten the required time of consolidation. In addition, dual function geosynthetic had been proposed to prevent excavation slope failures caused by the drawdown of water level along irrigation/drainage canals.

Keywords: Prefabricated vertical drain; Vacuum-assisted preloading; Groundwater recharge; Electro-conductive PVD; Staged preloading; Dual function geosynthetic

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1. Introduction

The low strength and high compressibility characteristics and, in several areas, the general subsidence caused by the excessive deep well pumping of the soft Bangkok clay deposits are the major problems that the geotechnical engineer has to consider when choosing the most appropriate ground improvement scheme for a particular project. The average ground subsidence recorded in 1990 was about 20–40 mm/yr (Bergado et al., 1990); however, some areas subsided as much as 100–150 mm/yr (Bergado et al., 1999).

The current soil consolidation techniques utilized in improving the strength of soft Bangkok clay deposit are surcharge preloading with prefabricated vertical drain (PVD) and vacuum-assisted preloading with PVD. In the PVD method, since the pore water that has been squeezed out during the consolidation of the clay due to preloading can flow a lot faster in the horizontal direction towards the drain, advantage is taken of higher horizontal permeability of clay. Also, the installation of PVD in the clay reduces the length of drainage paths and, subsequently, the time to complete the consolidation process. In effect, PVD installation accelerates the consolidation process of the clay subsoil, hence rapid strength increase of the same results improving the stability of structures built on top of it. In addition, the combination of either vacuum preload or electro-osmotic (EO) consolidation with the PVD technique can further enhance the effectiveness of PVD. EO consolidation involves the application of direct current electricity through electro-conductive PVD electrodes in order to create electro-osmosis and, thus, remove faster the pore water from the clay. Vacuum-assisted preloading has been applied in full scale, while the possibility and practicability of implementing electo-osmotic (EO) consolidation has been studied so far only in the laboratory. Moreover, the installation of PVDs can also effectively recharge the ground and, therefore, erase the negative piezometric level in the subsoil that causes much subsidence in Bangkok soft clay. More recently, the innovative use of dual function geosynthetic, i.e., both drainage and reinforcement, has been proposed for the repair of slope failure along drainage canal (Bergado et al., 2000).

2. Consolidation of soft Bangkok clay using PVD

The soft Bangkok clay foundation at the site of the Second Bangkok International Airport (SBIA) was improved using preloading consolidation technique with PVD. The site is located 30 km southeast of Bangkok.

2.1. Subsoils at SBIA site

At the SBIA site, the soft clay layer, which is overlaid by 2 m thick weathered crust, was found to a depth of 12 m, as shown in Fig. 1. Also shown in the figure are data of water contents, Atterberg limits, and shear strengths with depth, respectively. The groundwater table varies from 0.5 to 1.0 m below the ground surface.

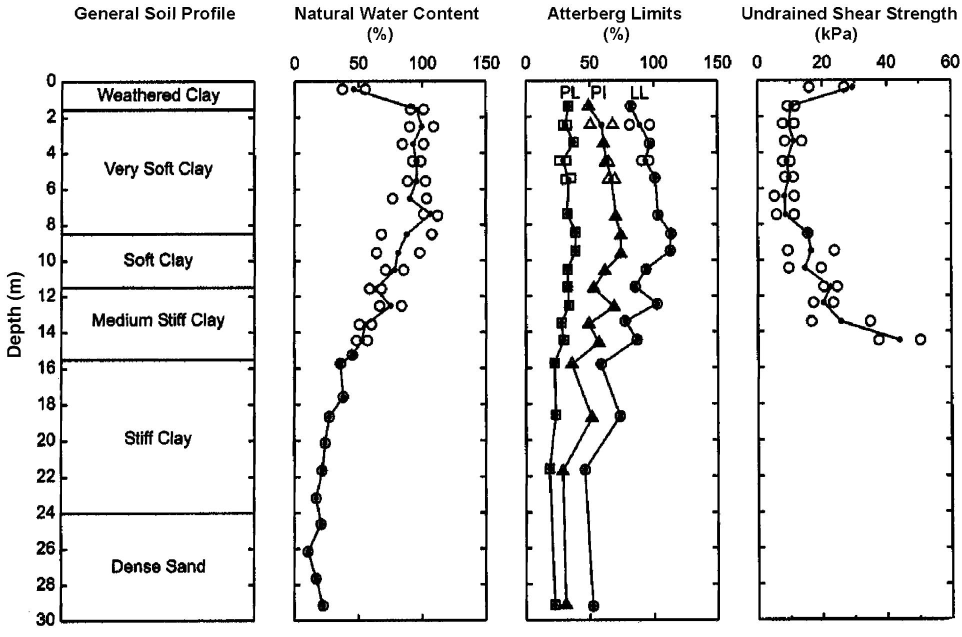

Fig. 1. Soil profile and properties at SBIA site.

2.2. PVD test embankments at SBIA

Three test embankments (TS1, TS2 and TS3) were constructed on PVD improved subsoil at SBIA site, with PVDs installed down to 12 m depth in a square pattern and spaced at 1.5, 1.2 and 1.0 m, respectively, in April 1994. The mandrel utilized during PVD installation was a rectangular cross-section having a thickness of 6 mm and outside dimension of 150 mm × 45 mm. Three different products of PVD, namely, Flodrain, Castle Board, and Mebra drain, were installed at TS1, TS2, and TS3, respectively. Both products have PVDs of equal dimension: 4 mm × 100 mm with equivalent diameter of 52 mm. The specification criteria for PVD at SBIA are summarized in Table 1. After PVD installation, the thickness of sand drainage was raised to 1.5 m from the previous 1.0 m thickness prior to the filling of embankment. Embankment filling was done in stages up to 4.2 m high (i.e., 75 kPa of surcharge) using clayey sand having compacted unit weight of 18 kN/m^3. For test embankment TS3: Stages 1,2,3 and 4 loading ended at 18, 45, 54 and 75 kPa, respectively. Moreover, a berm of 5 m wide and 1.5 m high was included when the surcharge increased from 54 to 75 kPa. Finally, each test embankment has 40 × 40 m in plan dimensions with 3H:1 V side slope and a finished height of 4.2 m.

2.3. Performance of test embankments at SBIA

The surface and subsurface settlements, lateral movements, and excess pore pressure in the three test embankments were measured by settlement plates,

Table 1
Specification criteria for PVD at SBIA

Properties	Test designation	Proposed values
Apparent opening size (μm)	ASTM D4751-87	<90
Grab tensile strength (kN)	ASTM D4632-91	>0.35
Trapezoidal tear strength (kN) (filter only)	ASTM D4533-88	>0.10
Puncture resistance (kN) (filter only)	ASTM	
D4833-88	>0.10	
Burst strength (kN/m^2) (filter only)	ASTM D3786-80a	>900
Discharge capacity at 7 days, 200 kPa at hydraulic gradient of 1 (m^3/yr)	ASTM D4716-87	>500
Discharge capacity at 200 kPa and hydraulic gradient of 1 (m^3/yr)	Modified triaxial (straight)	>500
Equiv. diameter = (length + width)/2 (mm)	—	>50
AOS/D_{85} (opening size of filter/grain size of clay)	—	>3

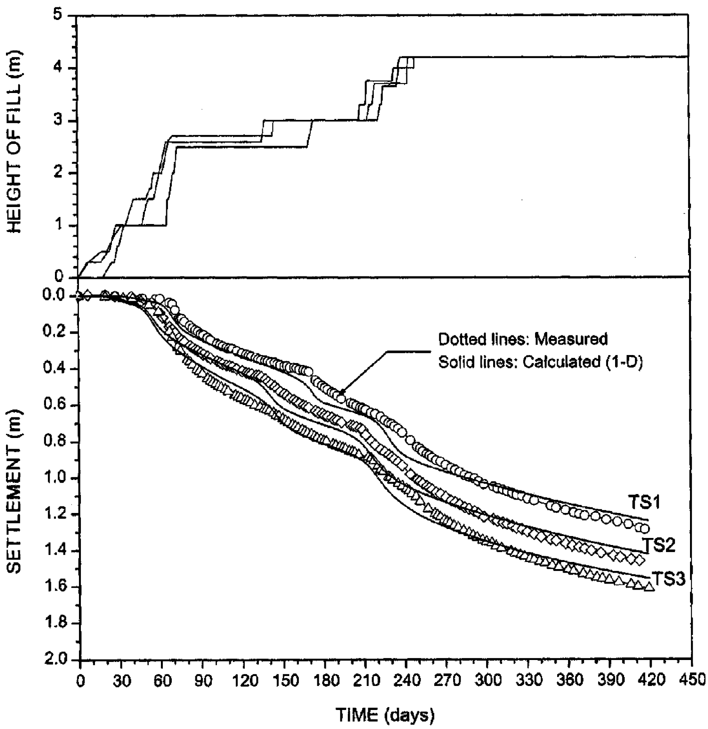

Fig. 2. Calculated and measured settlements of TS1, TS2 and TS3.

inclinometers and piezometers, respectively. Similar trends of settlements were observed in all three test embankments (Fig. 2). TS3 with the closest spacing of PVDs turned out to have the largest rate of settlement, while TS1 with the widest spacing of PVDs the smallest rate of settlement. Also shown in the figure are the

calculated settlements for the three test embankments using Terzaghi's one-dimensional consolidation theory. In addition, the measured pore pressure profile for TS3 is shown in Fig. 3. Fig. 3 shows that after 660 days of preloading (February 1996) the excess pore pressure had fully dissipated, thus consolidation of soil ended for less than 2 years inclusive of embankment construction. If there were no PVD, then the consolidation would have taken place until about 20 years. The data from the dummy piezometer, located at the unloaded part of the site and away from the influence of test embankments, are also incorporated in Fig. 3 (line ABC). This reading, which represented the initial pore pressure of the subsoil before preloading, indicated that the site has been experiencing piezometric drawdown from 8 m and below. The drawdown was an indication that the site is a subsiding ground. Nevertheless, the installation of PVD has recharged and eliminated the drawdown (line MNPQ) from 8 m down to 12 m depth at the bottom end of PVD. This observation is a substantiation that PVD installation can also effectively recharge the ground and, consequently, can eliminate the piezometric drawdown down to the bottom of PVD installation.

Furthermore, the measured undrained shear strength at the soft clay layer after 660 days of preloading increased to twice the corresponding initial values (Fig. 4). The predicted values (solid lines) were calculated using the SHANSHEP technique

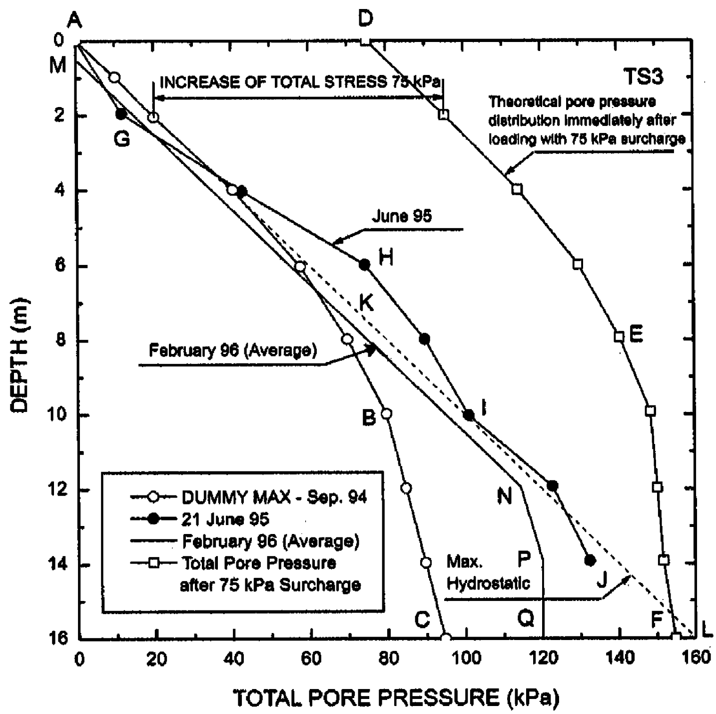

Fig. 3. Pore pressure profiles at test embankment TS3.

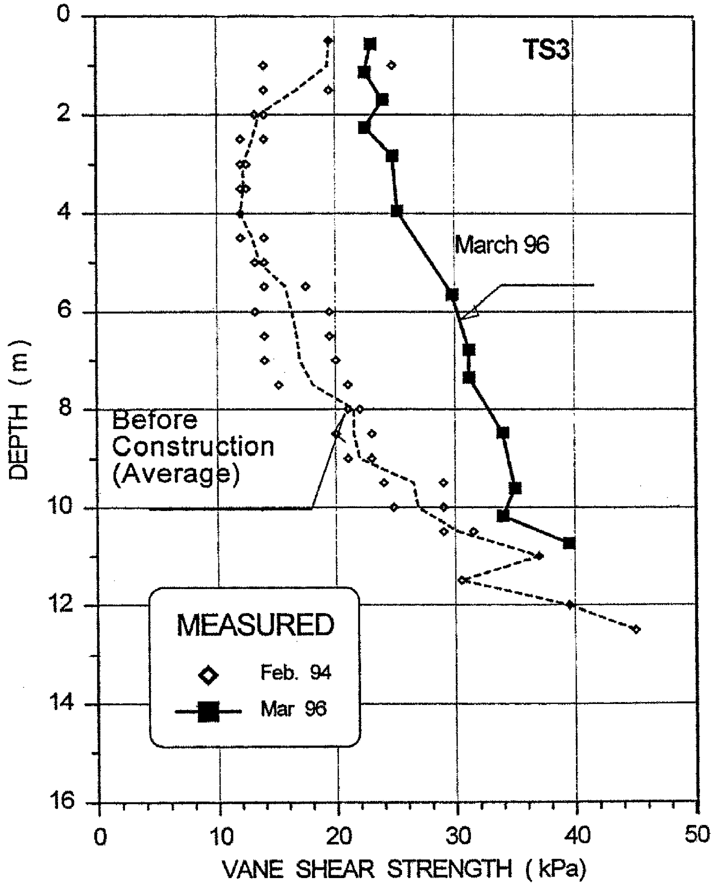

Fig. 4. Field vane shear strength measured in embankment TS3 at SBIA.

(Ladd, 1991) as follows:

$$\left(\frac{S_u}{\bar{\sigma}_{vo}}\right)_{OC} = \left(\frac{S_u}{\bar{\sigma}_{vo}}\right)_{NC} \mathrm{OCR}^m. \quad (1)$$

For soft Bangkok clay:

$$\left(\frac{S_u}{\bar{\sigma}_{vo}}\right)_{NC} = 0.22 \quad \text{and} \quad m = 0.8. \quad (2)$$

Hence, Eq. (1) becomes

$$\left(\frac{S_u}{\bar{\sigma}_{vo}}\right)_{OC} = 0.22(\mathrm{OCR})^{0.8}. \quad (3)$$

It is further shown in Fig. 4 that the predicted values of undrained shear strength yielded good agreement with the measured ones (Bergado et al., 2002a).

2.4. Back-analyzed compressibility and flow parameters from behavior of test embankments

The compressibility and flow parameters, namely, the horizontal coefficient of consolidation, C_h; the discharge capacity of PVD, q_w; and the ratio of the horizontal coefficient of permeability of undisturbed soil to the permeability of the disturbed soil within the smear zone, k_h/k_s, for the three test embankments were back-analyzed. Only results and relevant explanations will be presented here, for more details the reader is advised to refer to Bergado et al. (1996).

The graphical method of Asaoka (1978) in predicting final settlements based on observational procedure was utilized for the back-analyses of the above-mentioned parameters. By extending this method for the analysis of radial consolidation of soil with PVD, the following expression of the horizontal coefficient of consolidation, C_h, can be derived as

$$C_h = \frac{(1-\beta)D_e^2 F}{8\beta(\Delta t)}, \tag{4}$$

where β is the slope of the fitted line of the observed settlement points plotted at equal time interval, Δt, according to Asaoka's method; D_e is the equivalent diameter of a unit PVD influence zone; F is the factor which sums up the factors due to the PVD spacing (F_n), the smear effects (F_s) and the well resistance (F_r), are, respectively, defined as

$$F_n = \ln(D_e/d_w) - 0.75, \tag{5}$$

$$F_s = (k_h/k_s - 1)\ln(d_s/d_w), \tag{6}$$

$$F_r = \pi z(L - z)k_h/q_w. \tag{7}$$

Eqs. (5)–(7) are based on the modified solution of Hansbo (1979) for PVD applications derived from the general expression of average degree of horizontal consolidation, U_h, of sand drains which was previously developed by Barron (1948). The modified expression of U_h is

$$U_h = 1 - \exp\left(\frac{-8T_h}{F}\right), \tag{8}$$

$$T_h = \frac{C_h t}{D_e^2}, \tag{9}$$

where d_s is the diameter of the disturbed zone around the drain, d_w is the equivalent diameter of PVD, k_h is the horizontal permeability of the soil in the undisturbed zone, k_s is the horizontal permeability of the soil in the smeared zone, z is the distance of the point considered from the drainage end of the PVD, L is equal to twice the length of PVD for drainage boundary at one end only of PVD and is equal

to the length of PVD for drainage boundaries at both ends of PVD, and q_w is the discharge capacity of PVD at hydraulic gradient of unity.

Assuming the vertical compression of the soil is due mainly to the squeezing out of pore water through horizontal drainage towards the drain, the following expression of k_s can be derived:

$$k_s = m_v C_h \gamma_w. \tag{10}$$

Substituting of Eqs. (5)–(7) and Eq. (10) into Eq. (4), the following simplified expression can be obtained:

$$C_h = \frac{F_n + F_s}{C_1 - (C_2/q_w)}, \tag{11}$$

where

$$C_1 = \frac{8\beta(\Delta t)}{(1-\beta)D_e^2}, \tag{12}$$

$$C_2 = \pi z(L - z)m_v\gamma_w. \tag{13}$$

Since Eq. (11) consists of four unknowns, namely, k_h/k_s, d_s/d_w, q_w and C_h, the accuracy of the back-calculated values of C_h will be dependent on the correctness of the values of the others. However, this intricacy was diminished following the suggestion of Hansbo (1987) that d_s can be taken as twice the equivalent diameter of mandrel, d_m. This fairly reasonable correlation was also confirmed for soft Bangkok clay by Bergado et al (1991). Therefore, it is possible to obtain the relationship between C_h and q_w for certain values of smear ratio, k_h/k_s. Such relationships corresponding to actual settlement behavior of test embankments TS1, TS2 and TS3 are shown in Figs. 5–7, respectively. These figures indicate that the calculated C_h values become almost independent of q_w when the values of the latter exceed

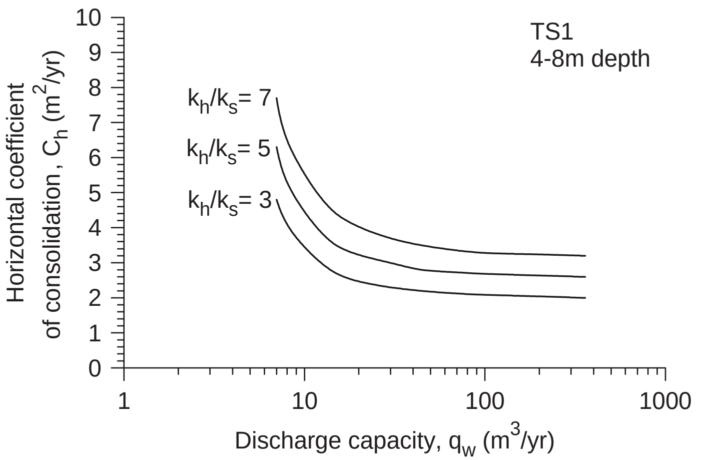

Fig. 5. Back-calculated $C_h - q_w$ relationship for TS1 test embankment.

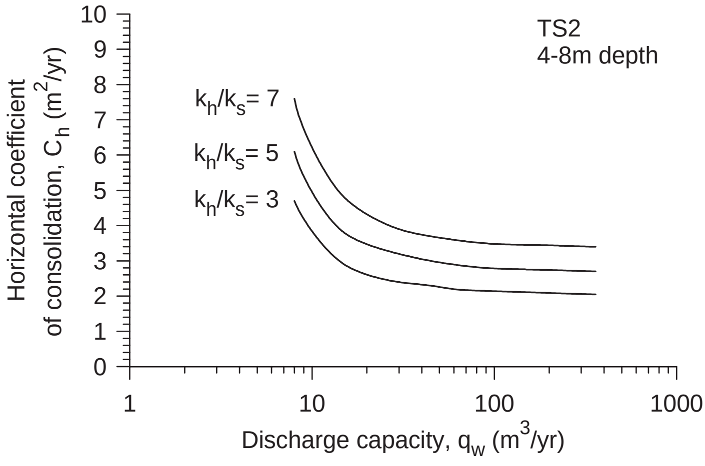

Fig. 6. Back-calculated $C_h - q_w$ relationship for TS2 test embankment.

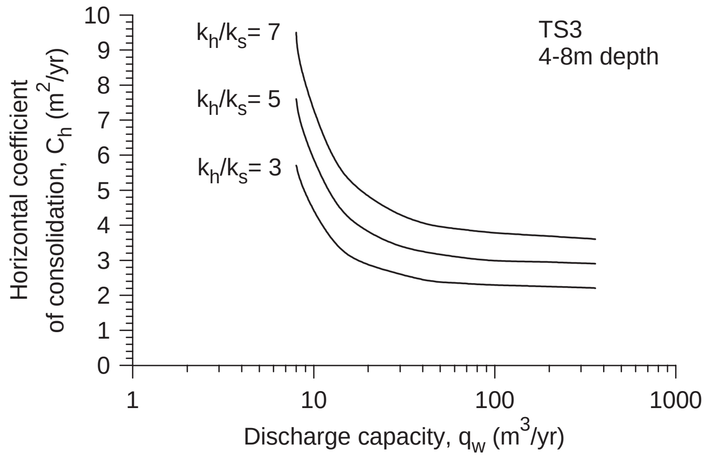

Fig. 7. Back-calculated $C_h - q_w$ relationship for TS3 test embankment.

$30\,m^3/yr$. It can be seen further on these plots that C_h is sensitive to the ratio k_h/k_s, which accounted for the soil disturbance at the smeared zones, for all possible values of q_w. Consequently, the underestimation of smear effects and/or overestimation of the discharge capacity will lead to underestimation of the C_h value.

Assuming $k_h/k_s = 5$, $d_s/d_m = 2$ and $q_w = 30\,m^3/yr$, together with the calculated values of β, the average C_h values for depth interval 0–12 m can be obtained, and are tabulated in Table 2. The corresponding C_h values for the actual soil profile at TS2 embankment are given in Table 3. Values of C_h in Table 3 can be compared directly with the corresponding field measured values, which were obtained by piezocone

Table 2
Calculated values of β, F and C_h for depth interval of 0–12 m from Asaoka (1978) method

Embankment	β	F	C_h (m^2/yr)
TS1	0.865	6.24	4.2
TS2	0.800	5.98	4.1
TS3	0.725	5.77	4.2

Table 3
Back-calculated values of C_h for subsoils under TS2 embankment

Subsoil layer	Weathered crust	Soft to very soft clay	Soft to medium clay
Depth (m)	0–2	2–8	8–12
C_h (m^2/yr)	4.0	3.0	7.8

Table 4
Values of C_h from piezocone tests (m^2/yr)

Depth (m)	PC-1	PC-2	PC-3
4	3.3	4.4	3.8
8	4.3	4.7	4.2
12	8.8	7.9	7.1

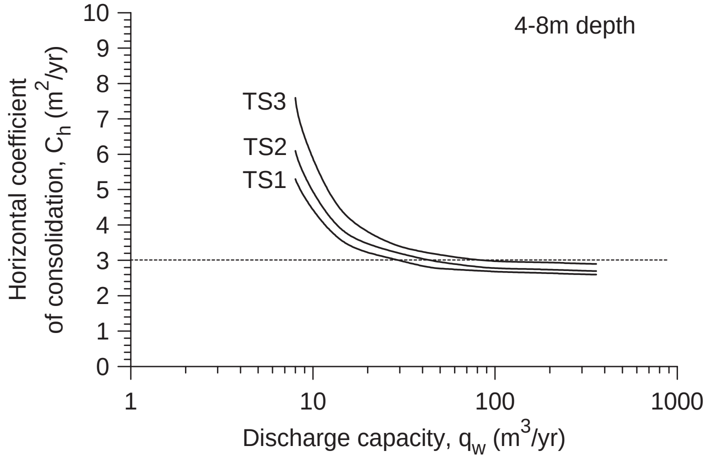

Fig. 8. Comparison of $C_h - q_w$ relationship for the three test embankments ($k_h/k_s = 5$).

tests, given in Table 4. There is good agreement between calculated and measured values of C_h. Moreover, the comparison of $C_h - q_w$ plots for each of the three test embankments, as shown in Fig. 8, using $k_h/k_s = 5$ and $C_h = 3\,m^2$/yr obtained from the piezoprobe tests of Moh and Woo (1987) for the soft clay layer from 4 m down to 8 m depth yielded reasonable range of values of q_w of about 30–90 m^3/yr.

2.5. Current recommended values of discharge capacity

A wide range of discharge capacity, q_w, values have been specified by several authors for proper functioning of vertical drains. A summary of these values is given in Table 5. The back-calculated values of discharge capacity obtained from test embankments TS1, TS2 and TS3 are at the lower bound of values in Table 5. As estimated from field measurements, the values of maximum flow rates varied from 7.88 (De Jager and Oostveen, 1990) to 52.56 m^3/yr (Lawrence and Koerner, 1988). Therefore, the back-analyzed discharge capacity values from test embankments TS1, TS2 and TS3, as discussed in the preceding section, have confirmed and agreed within the range of values from these investigators.

2.6. Effect of variable preloading on the rate of consolidation of PVD improved ground

The Landside Road System of SBIA Project was improved with PVDs down to 10 m depth. PVDs were installed at 1.0 m spacing in triangular patterns. There were three different types of surcharge loading, namely, 45, 71 and 91 kPa corresponding to Type I, Type II and Type III embankments, respectively. However, only Type II will be presented here (Fig. 9).

The actual measured construction settlement has been simulated numerically using PVD-SD Version 2.3 computer software, which is a one-dimensional finite element method (FEM), in order to back-calculate the horizontal coefficient of consolidation, C_h, of the underlying soft clay deposit from 1.5 m down to 10 m depth. The results of the simulation are shown in Fig. 10. The corresponding C_h values utilized during the analysis are summarized in Table 6. The good agreement of the calculated settlements with measured values (Fig. 10) indicated that the rate of consolidation of the foundation soft clay, and hence the values of C_h, are in fact very much dependent on the currently applied intensity of preload. As the preload increased, the coefficient of horizontal consolidation C_h decreases.

Table 5
Current recommended values for specification of discharge capacity (Bergado et al., 1996)

Sources	Values (m^3/yr)	Lateral stress (kPa)
Jamiolkowski et al. (1983)	10–15	500–300
Den Hoedt (1981)	95	50–300
Kremer et al. (1982)	256	100
Kremer (1983)	790	15
Hansbo (1979)	50–100	Not given
Rixner et al. (1986)	100	Not given
Van Zanten (1986)	790–1580	150–300
Holtz et al. (1989)	100–150	500–300
Lawrence and Koerner (1988)	150	Not given
Koda et al. (1984)	100	50
De Jager and Oostveen (1990)	315–1580	150–300

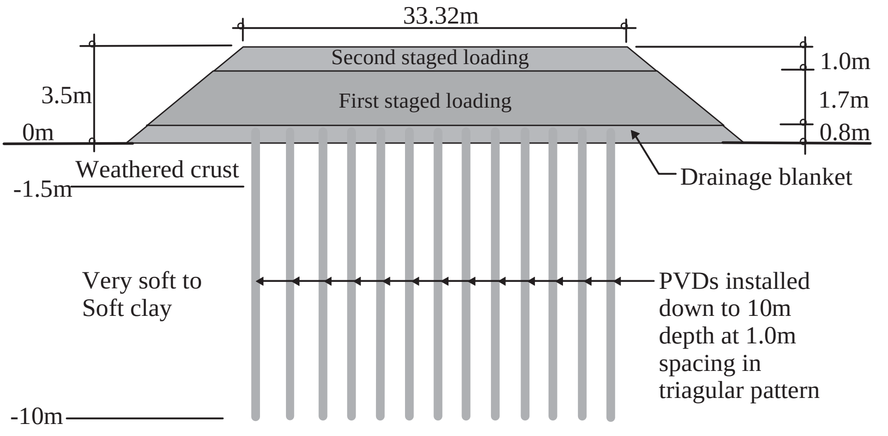

Fig. 9. Section of type II embankment at the Landside Road System of SBIA project.

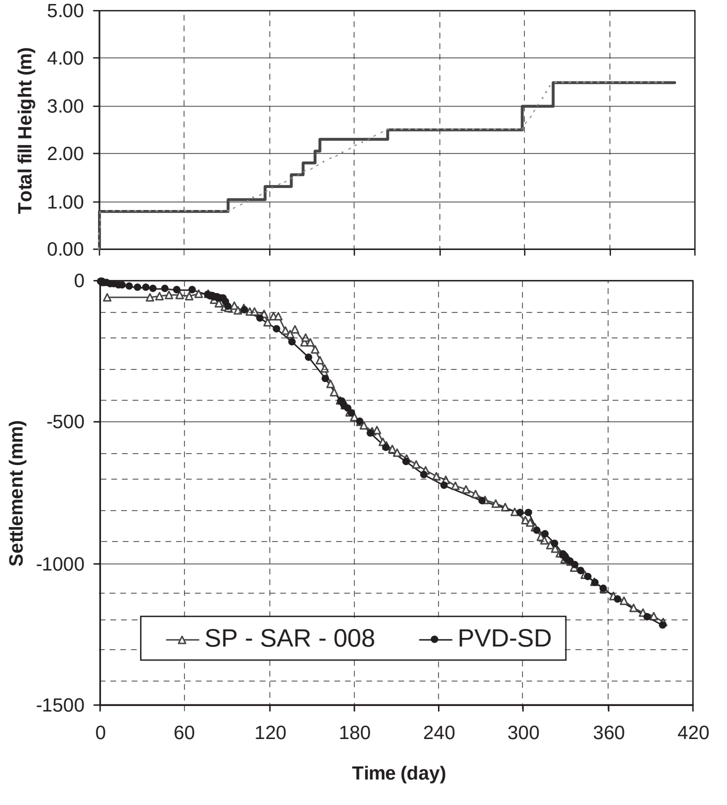

Fig. 10. Surface settlement of Type II embankment at the Landside Road System at SBIA.

Table 6
Back-calculated coefficient of horizontal consolidation, C_h, of the soft clay layer at SBIA site

Fill height (m)	Period after PVD installation (days)	C_h(m^2/yr)
0.80	—	—
0.80 with PVD	15–120	8.76
2.5 with PVD	120–210	6.57
3.5 with PVD	210–320	3.65

Moreover, the above-mentioned behavior of C_h can be attributed to the fact that during the preloading process a consequent reduction of void ratio occurred, which caused an exponential reduction of the permeability of the soil. If the coefficient of volume change (m_v) can be assumed not to vary much within the range of the applied loading, then the coefficient of consolidation of the soil can be expected to decrease with decreasing values of permeability (similar relationship as in Eq. (10)).

3. Vacuum-assisted preloading with PVD

Vacuum-assisted consolidation preloads the soil by reducing the pore pressure while maintaining constant total stress. This technique helps to shorten the time of preloading and to reduce the surcharge fill height.

Two additional 40 × 40 m embankments were constructed at the SBIA site close to the previous PVD embankment with a platform of 0.3 and 0.8 m sandfill for Embankments 1 and 2, respectively. Embankment 1 (TV1) utilized 12 m long PVD in conjunction with hypernet and nonwoven geotextile drainage system at the top of the PVDs. Embankment 2 (TV2) utilized 12 m long PVD together with corrugated pipe and nonwoven geotextile drainage system at the top of the PVDs. PVDs on both embankments were spaced at 1.0 m in triangular pattern. Furthermore, to maintain airtighness during vacuum operation, a very low density geomembrane liner (VLDPE) was placed on top of the drainage layer with both ends placed on the bottom of a perimeter trench and covered with 300 mm layer of sand-bentonite mix and water. After vacuum pumping for 45 days using –70 kPa and 100 m^3/h capacity vacuum pump, the embankments were raised in stages up to a height of 2.5 m, with the pump running continuously for 5 months (Bergado et al. 1998). However, even though a vacuum preloading of 75 kPa was anticipated, the actual measured values seemed to indicate an efficiency of only 40–50%, which is equivalent to surcharge pressure of 35–40 kPa.

The final settlement of TV1 and TV2, attained for about 5 months of pumping, were 0.74 and 0.96 m, respectively. Furthermore, the performance of TV2 when compared to previous studies using conventional sand surcharging produced acceleration at the rate of settlement by about 60% and reduction in the period of preloading by about 4 months, as shown in Fig. 11.

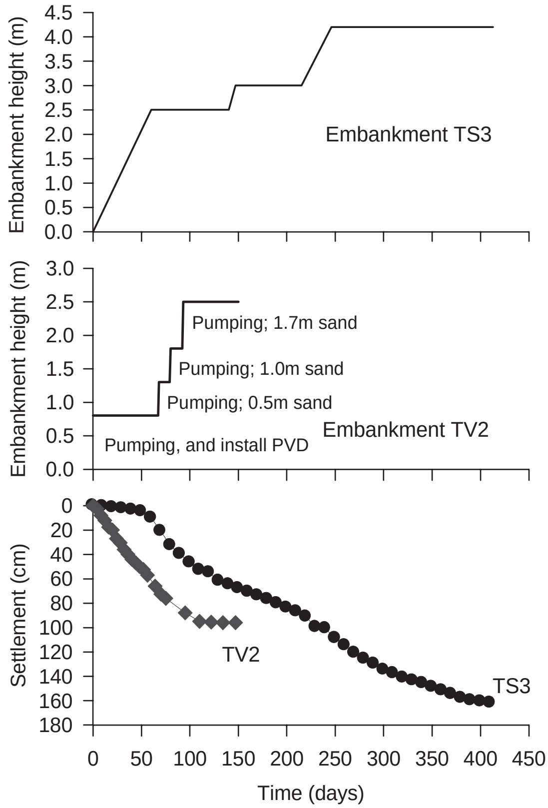

Fig. 11. Comparison of consolidation settlement between TV2 and TS3.

4. EO consolidation

4.1. Introduction

Electro-osmosis is the process wherein positively charge free water in a clay–water system moves from the anode towards the cathode. Upon application of direct current, cations in the diffused double-layer of water move toward the cathode to gain electrons and thereby become discharged. As the cations move, they carry with them porewater so that there is a new movement of water toward the cathode. If this porewater is removed at the cathode but not replaced at the anode, then consolidation of the soil will occur in an amount equal to the volume of water being removed. Furthermore, in order to balance the reduction of water in the soil surrounding the two electrodes, polarity is reversed at appropriate time interval.

The theory on EO has been around since 1897 by Helmholtz (1897), which was refined to form the known "Helmholtz–Smoluchowski theory." This theory is one of the earliest and most widely accepted until the present. In this model, the coefficient

of EO hydraulic conductivity through a porous medium, k_e ($m^2/s\,V$), which is a soil property that indicates the hydraulic flow velocity under a unit electrical gradient, is expressed as (Mitchell, 1993)

$$k_e = \frac{\zeta\varepsilon}{\mu} n, \tag{14}$$

where ζ is termed as zeta potential, ε (F/m) is the permittivity of the pore fluid, μ (Ns/m^2) is the viscosity of the pore fluid, and n is the porosity of the porous medium. For a sectional area, A, normal to the flow direction, the EO flow rate of pore water is given as

$$q_A = k_e i_e A, \tag{15}$$

where i_e is the electric potential gradient $\Delta E/\Delta L$.

4.2. Governing equation for consolidation by electro-osmosis between plate electrodes

For one dimensional flow between plate electrodes, the flow rate per unit cross-sectional area is given by the following equation (Mitchell, 1993):

$$q_h = -\frac{k_h \partial u}{\gamma_w \partial x} - k_e \frac{\partial V}{\partial x}. \tag{16}$$

Introduction of Eq. (16) in place of Darcy's law in the derivation of the diffusion equation governing consolidation in one dimension leads to

$$\frac{\partial^2 u}{\partial x^2} + \frac{k_e}{k_h}\gamma_w \frac{\partial^2 V}{\partial x^2} = \frac{1}{c_v}\frac{\partial u}{\partial t}, \tag{17}$$

where $u(x, t)$ is the excess pore water pressure at time 't' at a point located 'x' distance from the cathode, and $\partial V/\partial x$ is the voltage gradient at the same point. When equilibrium condition is reached, that is, when at any point the counter flow $(k_h/\gamma_w)(\partial u/\partial x)$ exactly balances the EO counter flow $k_e(\partial V/\partial x)$, consolidation is complete. At such a time, the flow rate q_h in Eq. (16) is nil, and the pore pressure at equilibrium at any point is given by

$$u(x, \infty) = -\frac{k_e}{k_h}\gamma_w V. \tag{18}$$

4.3. Laboratory investigation

Studies on the effect of EO consolidation on soft Bangkok clay had been performed in the laboratory (Bergado et al., 2000). Another series of tests, which aimed to investigate and make confirmation of the former, was conducted (Sasanakul, 2000; Phothiraksanon, 2001). Since the test results are similar, only the latter test results will be explained here.

Undisturbed sampling using 250 mm diameter sampler was conducted to obtain samples of typical Bangkok clay at a site within the campus of Asian Institute of Technology (AIT). Using the extruding machine, the soil, with its top surface

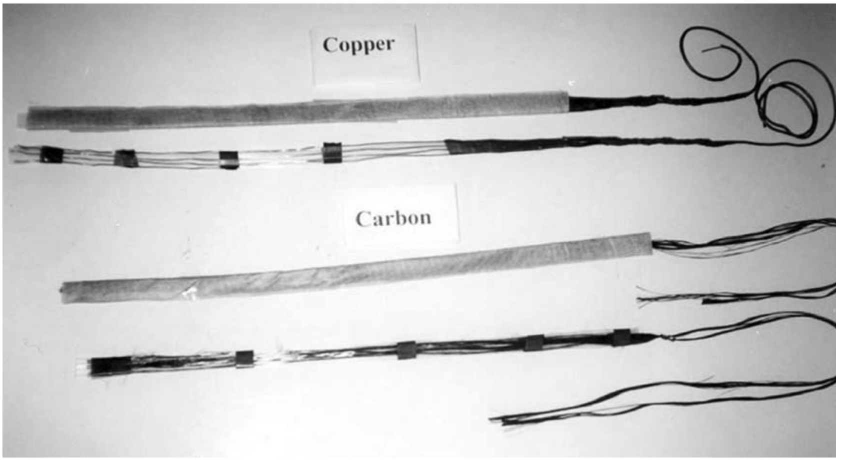

Fig. 12. Electro-conductive PVD.

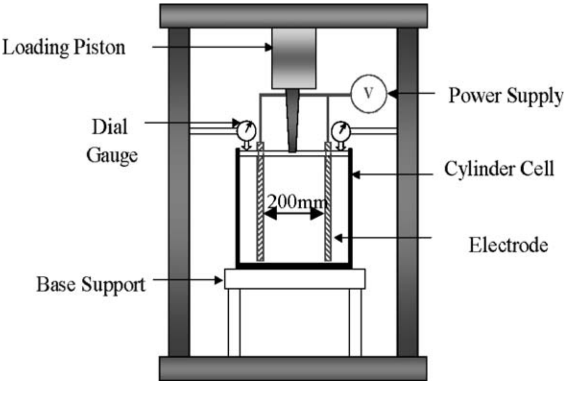

Fig. 13. Consolidometer cell.

trimmed smoothly, was pushed slowly into the cell having diameter of 230 mm and height of 300 mm until it touches the bottom of the cell. Two types of electro-conductive drains made from common PVD were used. These consisted of copper electrodes made by inserting 2 mm diameter copper rods into the drain core, and carbon electrodes made by inserting carbon fibers (Fig. 12). The finished electro-conductive drain has a width of 20 mm. Two electro-conductive drains were inserted at 200 mm apart per cell (Fig. 13). Geotextile filter was provided at the top of the specimen. However, both geotextile and top cap were provided with holes 200 mm apart to allow some portion of the electro-conductive drains to come out for power supply connection of the electrodes. Moreover, vertical stress of 5 and 50 kPa, voltage gradients of 60 and 120 V/m, and polarity reversal every 24 h were the conditions set during the subsequent series of consolidation tests.

The experimental set-up is shown in Fig. 13. The apparatus consisted of the loading piston and the cylinder cell. The loading piston was connected to an air compressor which was provided with a pressure regulator valve. Each test was stopped when the soil reached 90% consolidation. More details of the test program can be referred to in the literature (Bergado et al., 2000).

4.4. Effect of electrode type and voltage gradient

The result revealed that larger settlements and higher shear strengths were obtained on samples consolidated with electro-conductive PVDs compared to that with ordinary PVDs only (Figs. 14 and 15). At higher voltage gradient, faster rate of consolidation and higher magnitude of settlements were achieved compared to lower voltage gradient. The carbon electrodes displayed better results compared to copper electrodes in either voltage gradients, which conformed to the previous study of Bergado et al. (2000). Moreover, the shear strengths between the anode and the cathode were almost equal (Fig. 15), indicating the effectiveness of 24-h polarity reversal.

4.5. Effects of preloading with and without EO treatment on the compressibility and permeability of the resulting improved clay

Oedometer tests were conducted on EO consolidated soft Bangkok clay samples taken at SBIA site. Prior to oedometer testing, undisturbed sample was first subjected to EO consolidation inside 230 mm diameter cylinder cell using carbon electrodes with the following conditions: voltage gradient of 60 V/m, polarity reversal every 24 h, and applied vertical stress (preload) of either 7.5 or 10 kPa. After

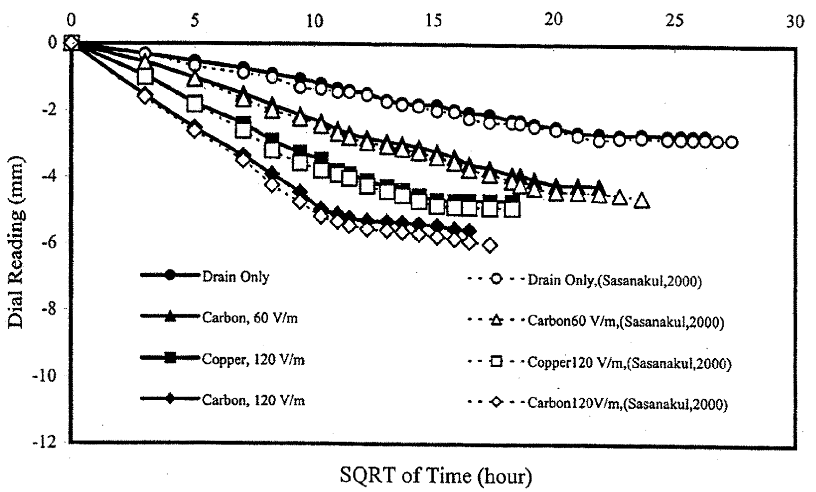

Fig. 14. Settlement versus time curve.

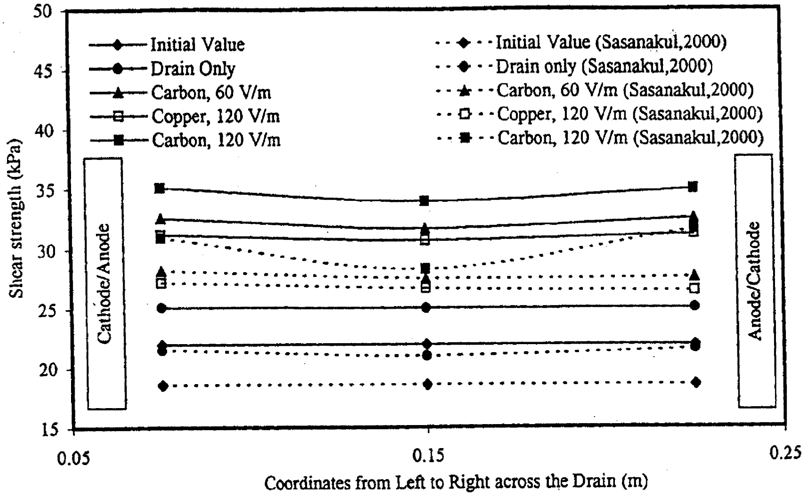

Fig. 15. Shear strength.

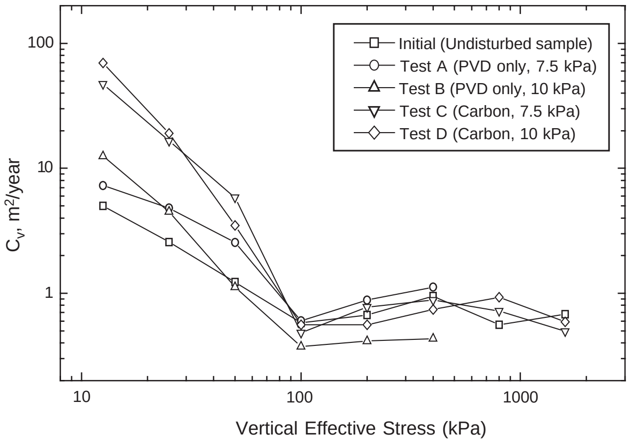

Fig. 16. Coefficient of vertical consolidation versus effective vertical stress.

EO consolidation, undisturbed samples were extracted from each cylinder cell for oedometer tests (Hormdee, 2002).

The coefficient of consolidation (C_v), the coefficient of permeability (k_v) and the mean compression indices (C_c) were calculated from the results of oedometer tests. At stress levels below the maximum past pressure of the base clay, the coefficients of vertical consolidation were found to be relatively high; the higher the preload during EO consolidation, the higher the C_v values of the resulting consolidated samples (Fig. 16). Significant increase of C_v was observed on preconsolidated samples with electro-conductive drains compared to those preconsolidated with ordinary PVD

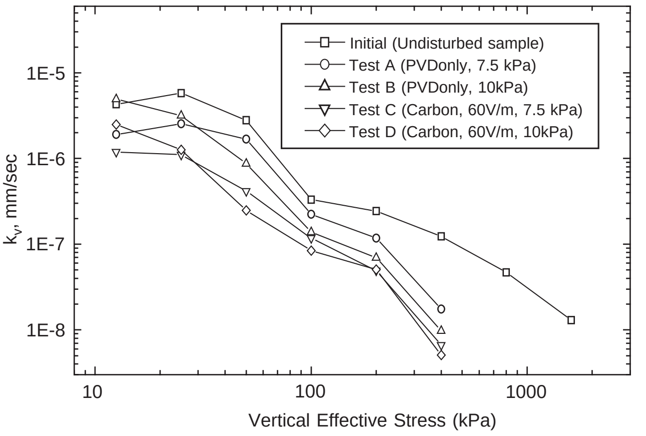

Fig. 17. Coefficient of vertical permeability versus effective vertical stress.

Table 7
Compression indices of specimens with and without electro-osmotic consolidation

Sample type	C_c
Undisturbed sample	1.10
Test A (PVD only, 7.5 kPa)	1.08
Test B (PVD only, 10 kPa)	0.99
Test C (carbon electrodes, 7.5 kPa)	0.93
Test D (carbon electrodes, 10 kPa)	0.73

only. However, the C_v decreased dramatically with increasing effective vertical stress and settled almost constant (about 1 m^2/yr) at stress levels beyond the maximum past pressure of the base clay. The coefficient of permeability, k_v, decreased with increasing effective vertical stress due to the subsequent decrease of the void ratio (Fig. 17). Although this trend of k_v occurred regardless of the type of improvement of the samples, the magnitude of k_v, however, at certain effective vertical stress level is higher for samples preconsolidated with ordinary PVD only than those preconsolidated with electro-conductive drains. Much higher k_v values were obtained on undisturbed clay samples. Thus, EO consolidation caused the k_v of clay to decrease, due to the additional consolidation of the soil as a result of EO effect. Moreover, the higher the preload during EO consolidation, the lower the values of k_v of the resulting consolidated soil. In addition, the mean compression indices, C_c, tended to be lower for samples preconsolidated with electro-conductive drains (Table 7). This trend of C_c is consistent with the trends of C_v and k_v. From the general relationship $k_v = C_v \gamma_w m_v$, if C_v stays constant (at stress level beyond the maximum past pressure) regardless of the degree of treatment of the samples (Fig. 16), and if k_v decreases with increasing degree of treatment (Fig. 17), then the

coefficient of volume compressibility, m_v, should decrease with increasing degree of treatment. If m_v decreases, C_c decreases as well. This behavior of C_c is attributed to the consequent further reduction of void ratio after preloading due to the additional consolidation of the soil resulting from EO consolidation.

5. Groundwater recharge using PVD

As discussed in Section 2.3, the installation of PVD in test embankment TS3 at SBIA has caused recharge in the underlying subsoil and, eventually, erase the piezometric drawdown up to a depth of PVD installation (Fig. 3). In order to have further understanding to this eventuality of using PVD for recharge of piezometric level, another test embankment was constructed in November 1999 at the campus of AIT. The $20 \times 25 \times 1.2$ m sand embankment, which served as drainage blanket and construction platform at the same time, was constructed on ground improved by 10 m depth PVD spaced at 1.0 m in square patterns. In the PVD area, water was sprayed for 6 h everyday in order to simulate rainfall and to recharge the drawdown of piezometric level. The pore pressures, measured from open standpipe piezometers installed at different depths, were monitored for a period of 4 months.

As measured from the dummy area, the pore pressure profile with depth indicated that the piezometric drawdown was observed from 5 m depth downwards (Fig. 18). However, in the PVD area, the piezometric level was located above the hydrostatic level. As shown in Fig 19, the piezometric drawdown had disappeared. This result substantiated the previous finding in test embankment TS3 at SBIA. Therefore, PVDs can be effective channels to recharge the ground, and erase the piezometric level drawdown to the depth of PVD installation (Bergado and Lorenzo, 2002).

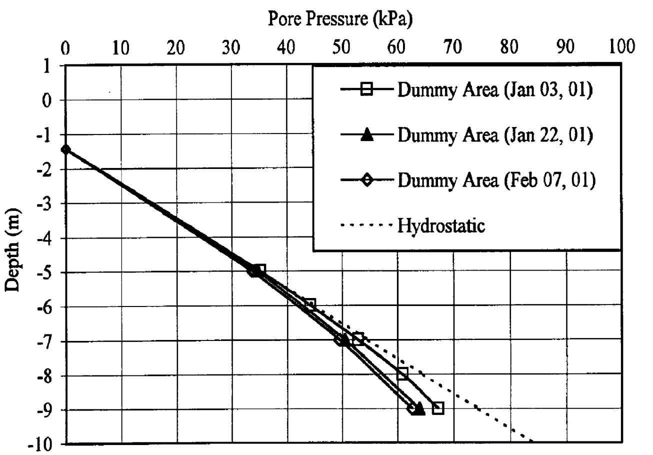

Fig. 18. Piezometric pressure in the dummy area at AIT campus.

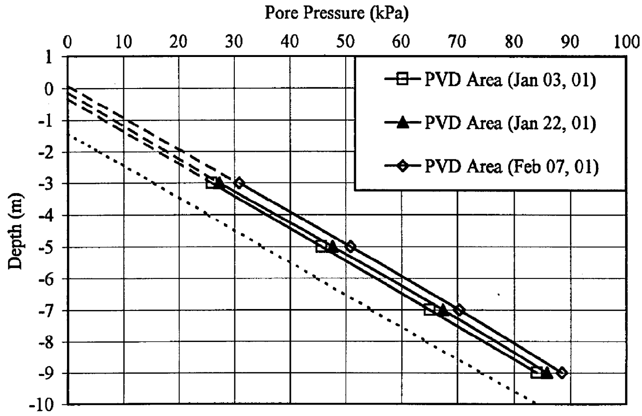

Fig. 19. Pore pressure with depth in the PVD area at AIT campus.

6. Dual function geosynthetic

Recent innovation of soil reinforcement for slope protection utilizes the versatility of dual function geosynthetic. Strictly, the term 'dual function' used herein includes only drainage and reinforcement functions. Moreover, the dual function geosynthetic studied in this paper refers to the high strength nonwoven/woven geotextile only.

Slope failures, caused by the drawdown of water level in an irrigation/drainage canal, occurred at Klong 15, 28 and 31 in Nakornnayok Province, Thailand. The deep mixing method (DMM) soil-cement columns had been implemented to solve a such problem. However, due to its high-cost, the search for a cost-effective alternative has been undertaken. The possibility of using dual function geosynthetic has been studied (Bergado et al., 2002b). The actual slope protection scheme using DMM soil-cement columns is shown in Fig. 20, and the alternative scheme using dual function geosynthetic is given in Fig. 21. The generalized soil profile at the site is summarized in Table 8. For the original scheme, soil-cement piles, with 0.60 m diameter and with 500 kPa UC strength, were installed down to 8 m depth. For the alternative scheme, dual function high strength nonwoven/woven geotextiles were utilized. In the latter scheme, two modes of slope instability were analyzed, namely, Case A with limitation of foundation deformation, and Case B with allowable large settlement. In Case A, the mobilized tensile force in the geotextile was calculated based on the critical strain (ε_c), which is about 3%, at the initiation of bearing failure of foundation soil. In Case B, the mobilized tensile force was calculated based on the critical strain (ε_c) plus the localized strain (ε_{lc}) associated with the development of slip surface at the onset of failure caused by the reorientation of reinforcement tension force (Long et al., 1997, Bergado et al., 2003). Furthermore, the results of the stability analyses using SLOPE/W program are tabulated in Table 9. The factor of safety of 1.972 was obtained for the scheme using DMM columns. From Table 9, the scheme with four layers of reinforcements in the embankment and three layers in the

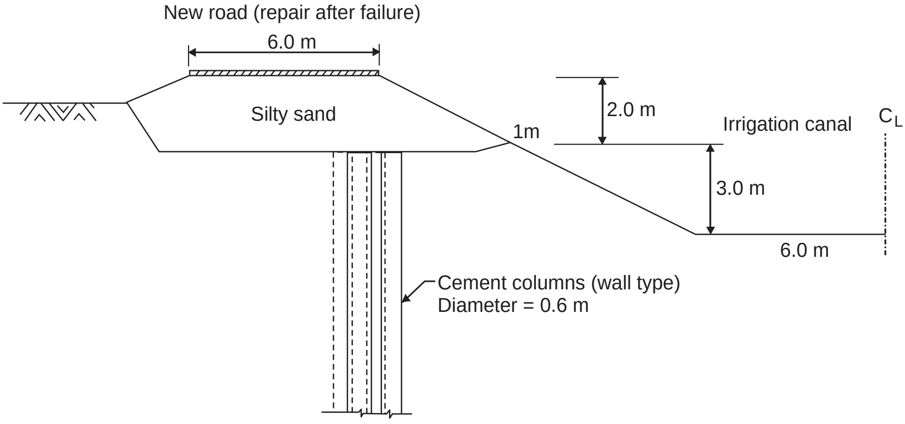

Fig. 20. Use of DMM columns for the repair of irrigation canal at Klong 15, 29 and 31.

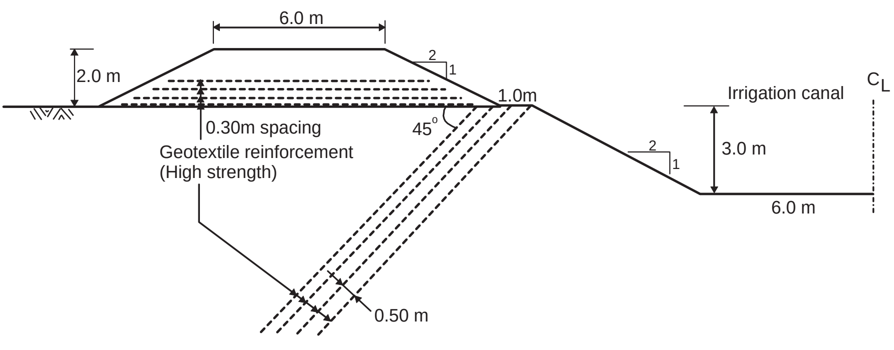

Fig. 21. Slope reinforcement scheme using dual function high strength nonwoven/woven geotextile reinforcement.

Table 8
Parameters for limit equilibrium analysis

Soil condition	Depth (m)	γ_t (kN/m^3)	S_u (kPa)	ϕ' (Degrees)
Silty sand (embankment)		19.0		30
Weathered crust	0–2	17.5	25.0	—
Soft clay	2–16	15.0	12.5	—
Medium clay	16–22	16.0	25.0	—
Stiff clay	22–27	18.0	120.0	—

soft clay (Fig. 21) yielded the same stability performance as the DMM columns slope protection scheme. However, subsequent cost analysis revealed that utilizing geotextile reinforcements is cheaper by about half the cost of implementing DMM soil-cement columns (Bergado et al., 2002b).

Table 9
Factor of safety calculated by SLOPE/W

Description	FS (Case A)	FS (Case B)
Geotextile reinforcement		
(a) Inside embankment (2 layers)	0.983	1.022
(b) Inside embankment (4 layers)	1.022	1.260
(c) Inside embankment (2 layers) and soft clay (2 layers)	1.388	1.578
(d) Inside embankment (4 layers) and soft clay (2 layers)	1.662	2.020
(e) Inside embankment(4 layers) and soft clay (3 layers)	1.810	2.449
Soil-cement columns	FS = 1.972	

7. Conclusions

New innovative applications of PVD and dual function geosynthetics in soft Bangkok clay have been presented in this paper in addition to the commonly practiced versatility of PVD as artificial drainage path in the consolidation of clay soils.

The use of PVD in soil consolidation by surcharge preloading has been proven to effectively accelerate the consolidation of clay soil. Three full-scale tests embankments constructed at the SBIA site revealed that the 10 m thick soft Bangkok clay at the site reached its 90% consolidation within 1 year after embankment construction, for PVD spacing ranging from 1.0 to 1.5 m. Moreover, the compressibility and flow parameters have been successfully back-calculated from the behavior of the full-scale test embankment. The PVD installation also promoted uniform rate of consolidation and settlement.

On the other hand, back-analysis of the observed settlement of 3.5 m high embankment constructed on PVD improved ground at the Landside Road System of SBIA project revealed that each stage of preloading would correspond to certain magnitude of the horizontal coefficient of consolidation (C_h) of the underlying soft clay deposit. Calculated settlements were eventually found in good agreement with the measured ones when C_h values of 8.76, 6.57 and 3.65 m^2/yr corresponding to fill height of 0.8, 2.5 and 3.5 m, respectively, were utilized during the analysis. Moreover, the compression index (C_c) also reduced with increasing surcharge preloading levels.

Clay specimens improved with electro-conductive PVDs yielded an increase in the coefficient of consolidation and reduction of the compression index than those preloaded with ordinary PVD only, as obtained from oedometer tests. Moreover, the soft Bangkok clay with electro-conductive PVDs demonstrated faster rate of consolidation and higher magnitude of total settlement for the same amount of preload compared to one preloaded with ordinary PVDs only, as observed in the laboratory. Therefore, the advantages of the application of electro-conductive PVDs for soft clay improvement are not only to shorten the required preloading time but also to increase the rate of consolidation and decrease the compression index of the resulting improved soft clay foundation.

The combination of vacuum-assisted preloading further enhanced the performance of preloading with PVD. Vacuum-assisted preloading accelerated the rate of consolidation by 60% and subsequently reduced the required surcharge fill height, as observed in a full-scale embankment.

Furthermore, both test embankments at SBIA site and at AIT campus provided substantive evidence that PVD installation could recharge subsiding ground and, consequently, erase the piezometric drawdown to the level of PVD installation. Hence, it could minimize ground subsidence.

The use of dual function geosynthetics, performing both drainage and reinforcement functions, would lead to a more economical design in mitigating excavation slope failures at irrigation/drainage canals on soft ground.

References

Asaoka, A., 1978. Obsevational procedure for settlement prediction. Soils and Foundations 18 (4), 87–101.

Barron, R.A., 1948. Consolidation of fine-grained soils by drain well. Transactions of the ASCE 124, 709–739.

Bergado, D.T., Lorenzo, G.A., 2002. Recent developments of ground improvement in soft Bangkok clay. Proceedings of the International Symposium on Lowland Technology, September 2002, Saga University, Japan, pp. 17–26.

Bergado, D.T., Ahmed, S., Sampaco, C.L., Balasubramaniam, A.S., 1990. Settlement analysis of Bangna-Bangpakong highway on soft Bangkok clay. Journal of Geotechnical Engineering Division, ASCE 115 (1), 136–155.

Bergado, D.T., Asakami, H., Alfaro, M.C., Balasubramaniam, A.S., 1991. Smear effects of vertical drains on soft Bangkok clay. Journal of Geotechnical Engineering Division, ASCE 117 (10), 15–29.

Bergado, D.T., Long, P.V., Balasubramaniam, A.S., 1996. Compressibility and flow parameters from PVD improved soft Bangkok clay. Geotechnical Engineering 27 (1), 1–20.

Bergado, D.T., Chai, J.C., Miura, N., Balasubramaniam, A.S., 1998. PVD improvement of soft Bangkok clay with combined vacuum and reduced sand embankment preloading. Geotechnical Engineering Journal 29 (1), 95–122.

Bergado, D.T., Ruenkrairergsa, T., Taesiri, Y., Balasubramaniam, A.S., 1999. Deep soil mixing used to reduce embankment settlement. Ground Improvement 3, 145–162.

Bergado, D.T., Balasubramaniam, A.S., Patawaran, M.A.B., Kwunpreuk, W., 2000. Electro-osmotic consolidation of soft Bangkok clay with prefabricated drain. Ground Improvement 4, 153–163.

Bergado, D.T., Balasubramaniam, A.S., Fannin, R.J., Holtz, R.D., 2002a. Prefabricated vertical drains (PVDs) in soft Bangkok clay: a case study of the New Bangkok International Airport project. Canadian Geotechnical Journal 39, 304–315.

Bergado, D.T., Horpibulsok, S., Ngouchaurieng, P., 2002b. Innovative use of geosynthetics for repair of slope failures along irrigation/drainage canals on soft ground. Proceedings of the Seventh International Geosynthetic Conference (IGS-2002), Nice, France.

Bergado, D.T., Lorenzo, G.A., Long, P.V., 2003. LEM-back analysis of geotextile reinforced embankment on soft Bangkok clay—a case study. Geosynthetic International 9 (3), 217–245.

De Jager, W.F.J., Oostveen, J.P., 1990. Systematic quality control of vertical drainage. Proceeding of the Fourth International Conference on Geotextile, Geomembranes and Related Products, The Hague, pp. 321–26.

Den Hoedt, G., 1981. Laboratory testing of vertical drains. Proceeding of the 10th International Conference on Soil Mechanics and Foundation Engineering, Stockholm, Vol. 1, pp. 627–630.

Hansbo, S., 1979. Consolidation of clay by band-shaped prefabricated drains. Ground Engineering 12 (5), 16–25.

Hansbo, S., 1987. Design aspects of vertical drains and lime column installations. Proceedings of the Ninth Southeast Asian Geotechnical Conference, Bangkok, Thailand, pp. 8-1–8-12.

Helmholtz, H., 1897. Wiedemannus Annalen D. Physik 7, 137.

Holtz, R.D., Jamiolkowski, M., Lancellota, R., Pedroni, S., 1989. Behavior of bent prefabricated vertical drains. Proceeding of the 12th International Conference on Soil Mechanics and Foundation Engineering, Rio de Janeiro, Brazil, Vol. 3, pp. 1657–1660.

Hormdee, D., 2002. Effects of variable preloading on prefabricated vertical drain (PVD) improved ground with electro-osmotic consolidation and piezometric drawdown. M. Eng. Thesis No. GE-01-11. Asian Institute of Technology, Bangkok, Thailand.

Jamiolkowski, M., Lancellota, R., Wolski, W., 1983. Summary of discussion to specialty session 6. Proceeding of the Eighth European Conference on Soil Mechanics and Foundation Engineering, Vol. 3, Helsinki, Finland.

Koda, E., Szmanski, A., Wolski, W., 1984. Laboratory tests on geodrains durability in organic soils. Seminar on Laboratory Testing of Prefabricated Band-Shaped Drains, Milano, Italy.

Kremer, R., 1983. Discussion to specialty session 6. Proceedings of the Eighth European Conference on Soil Mechanics and Foundation Engineering, Vol. 3, Helsinki, Finland, pp. 1235–1237.

Kremer, R., De Jager, W., Maagdenberg, A., Mexvogel, I., Oostveen, J., 1982. Quality standards for vertical drains. Proceeding of the 2nd International Conference on Geotextiles, Las Vegas, USA, Vol. 2, pp. 319–323.

Ladd, C.C., 1991. Stability evaluation during staged construction. Journal of Geotechnical Engineering, ASCE 117 (4), 541–615.

Lawrence, C.A., Koerner, R.M., 1988. Low behavior of kinked strip drains geosynthetics for soil improvement. ASCE Geotechnical Special Publication 18, 22–35.

Long, P.V., Bergado, D.T., Balasubramaniam, A.S., 1997. Localized mobilization of geotextile reinforcement force at failure surface, Proceedings of the 14th International Conference on Soil Mechanics and Foundation Engineering, Hamburg, Germany, pp. 1761–1764.

Mitchell, J.K., 1993. Fundamentals of Soil Behavior, Wiley, New York, 437pp.

Moh, Z.C., Woo, S.M., 1987. Preconsolidation of Bangkok clay by nondisplacement sand drains and surcharge. Proceedings of the 9th Southeast Asian Geotechnical Conference, Bangkok Vol. 2, pp. 8-171–8-184.

Phothiraksanon, P., 2001. Effects of electro-osmotic consolidation of undisturbed and reconstituted Bangkok clay at low voltage gradient. Master Eng. Thesis, No. GE-00-19, Asian Institute of Technology, Bangkok, Thailand.

Rixner, J.J., Kraemer, S.R., Smith, A.D., 1986. Prefabricated vertical drains. Engineering Guidelines, Vol. 1. FHWA/RD-86/168, Federal Highway Administration, Virginia, 107pp.

Sasanakul, I., 2000. Electro-chemical changes in clay during electro-osmotic consolidation using copper and carbon electrode with PVD. Master Eng. Thesis No. GE-99-11. Asian Institute of Technology, Bangkok, Thailand.

Van Zanten, R.V., 1986. The guarantee of the quality of vertical drainage systems. Proceeding of the Third International Conference on Geotextile, Vol. 2, Vienna, Austria, pp. 651–655.